NISTIR 6288

NIST Workshop on Standards Development for the Use of Fiber Reinforced Polymers for the Rehabilitation of Concrete and Masonry Structures, January 7-8, 1998, Tucson, Arizona. Proceedings

Editor:
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Session secretaries:	Session chairs:	Session co-chairs:
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United States Department of Commerce
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ADVANCED COMPOSITES
TECHNOLOGY TRANSFER
CONSORTIUM

Report No.
ACTT–95/08

EARTHQUAKE RETROFIT OF BRIDGE COLUMNS WITH CONTINUOUS CARBON FIBER JACKETS

— Volume II, Design Guidelines —

by

Frieder Seible
M.J. Nigel Priestley
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Report to Caltrans, Division of Structures, prepared under the ARPA/TRP Program Agreement No. MDA 972-94-3-0030.

August 1995

Structural Engineering
University of California, San Diego
La Jolla, California

Advanced Composites
Technology Transfer Consortium

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1. INTRODUCTION

Recent earthquakes such as Whittier 1987, Loma Prieta 1989, and Northridge 1994, have demonstrated the vulnerability of older reinforced concrete bridge columns to fail under the imposed seismic deformation demands. Particularly vulnerable are bridge piers designed prior to the 1971 San Fernando earthquake since before that time the transverse or horizontal column reinforcement was only nominally provided, typically #4 bars (D 13 mm) placed 300 mm (12 in) on center, independent of column size, strength, or deformation demands. Even after 1971, substandard transverse reinforcement amounts and inadequate seismic reinforcement detailing can be encountered in some of the existing reinforced concrete bridge columns.

The functions of the transverse reinforcement are (1) to provide shear capacity to columns where principal tensile stresses cause inclined cracking, (2) to confine potential flexural plastic hinge regions for increased concrete deformation capacity and for lateral support to the longitudinal column reinforcement subsequent to cover concrete spalling, and (3) to clamp lap splices in the longitudinal column reinforcement. To fulfill these requirements, transverse reinforcement amounts can be calculated and designed based on established principles. Appropriate seismic detailing in the form of anchorage in the column core, welded hoops, or continuous spiral reinforcement will ensure functionality of this transverse reinforcement even subsequent to cover concrete spalling.

In existing reinforced concrete bridge columns where insufficient transverse reinforcement and/or seismic derailing are provided, three different types of failure modes can be observed under seismic load/deformation input.

The first and most critical failure mode is column shear failure (Fig. 1), where inclined cracking, cover concrete spalling and rupture or opening of the transverse reinforcement can lead to brittle or explosive column failures. The failure sequence consists of (1) the development of inclined cracks once the tensile strength of the concrete is exceeded, (2) the opening of inclined or diagonal cracks in the column and onset of cover concrete spalling, (3) rupture or opening of the transverse or horizontal reinforcement, (4) buckling of the longitudinal column reinforcement, and (5) disintegration of the column concrete core. While new column designs feature engineered and better detailed transverse or shear reinforcement, the shear strength of existing substandard columns can be enhanced by providing external shear reinforcement or strength to the column through carbon fiber wraps in the hoop direction. The shear capacity of columns needs to be checked both in the column end regions or potential plastic hinge regions and in the column center portion between flexural plastic and/or existing built-in column hinges.

The second column failure mode consists of a confinement failure of the flexural plastic hinge region (Fig. 2), where subsequent to flexural cracking, cover concrete crushing, and spalling, buckling of the longitudinal reinforcement, or compression failure of the core concrete initiate plastic hinge deterioration, associated with a shortening of the column in the plastic hinge zone. Plastic hinge failures typically occur with some displacement ductility, and are limited to shorter regions in the column. Thus, these failures are less destructive and, because of their large inelastic flexural deformations, are more desirable than the brittle column shear failures of the entire column as described above. This desired ductile flexural plastic hinging at the column ends can be achieved through added confinement in the form of increased hoop or transverse reinforcement in new and external jacketing in existing columns. The confinement objective is to prevent cover concrete spalling, to provide lateral support of the longitudinal reinforcement, and to enhance concrete strength and deformation capacities. All of these characteristics apply along the entire column perimeter and thus uniform confinement provided by circular hoops or a circular external jackets would be most beneficial. In rectangular columns, either a circular or oval jacket can provide confinement along the entire column perimeter while rectangular jackets effectively only provide inward corner forces, and significant jacket thickness needs to be provided between corners to restrain lateral dilation and column bar buckling. However, large scale tests (Fig. 3) have shown that appropriately designed rectangular carbon jackets can provide sufficient confinement and bar buckling restraint to achieve high flexural displacement ductility levels.

Finally, some existing bridge columns feature lap splices in the column reinforcement, which for ease of construction are located at the lower column end to form the connection between the footing and the column. Starter bars for the column reinforcement are placed during the footing construction and lapped with the longitudinal column reinforcement in this region of maximum column moment demand, i.e., the potential plastic hinge region. While the confinement concepts discussed above for plastic hinge regions also apply to lap-spliced column ends, the flexural strength of the column can only be developed and maintained when debonding of the reinforcement lap splice is prevented. Lap splice debonding occurs once vertical micro cracks develop in the cover concrete and debonding gets progressively worse with increased vertical cracking and cover concrete spalling, (Fig. 4). This flexural capacity degradation can occur rapidly at low flexural ductilities in cases where short lap splices are present and little confinement is provided, but can also occur more gradually with increased lap length and confinement. Confinement can again be provided by external jacketing or winding with continuous carbon fibers, where jackets with convex curvature are again more advantageous to provide continuous lateral clamping pressures to the column bar lap splices along the entire column perimeter

None of the above failure modes and associated column retrofits can be viewed separately since retrofitting for one deficiency may only shift the seismic problem to another location and failure mode, without necessarily improving the overall deformation capacity. For example, a shear critical column strengthened over the column center region with carbon wraps is expected to develop flexural plastic hinges at the column ends, which in turn need to be designed and retrofitted for the desired confinement levels. Furthermore, lap splice regions need not only be checked for the required clamping force to develop the capacity of the longitudinal column reinforcement, but also for confinement and ductility of the flexural plastic hinge.

Based on the different failure mechanisms discussed above different column regions which will require different jacket designs can be identified, as shown in Figs. 5 and 6, where L_s = lap splice length, L_{c1} = primary confinement region for plastic hinge, L_{c2} = secondary confinement region adjacent to plastic hinge, L_v = shear strengthening region, L_v^i = shear retrofit inside the plastic hinge zone and L_v^o = shear retrofit outside the plastic hinge zone. The secondary confinement region is necessary to prevent flexural plastic hinging above the primary plastic hinge zone when confinement allows for significant overstrength development in the primary plastic hinge. Plastic hinge confinement lengths L_{c1} and L_{c2} are tied to the column geometry based on the expected plastic hinge length both in terms of column depth or diameter in the loading direction, and to the shear span or distance from the column hinge to the point of contraflexure. The lap splice length L_s is directly defined by the lap length of the starter and column bars and the shear length L_v is taken as the remaining region between the previously defined end zones. In order to avoid direct contact between thick column end jackets and the adjacent bridge footing or cap-beam. a gap is designed to allow plastic hinge rotation without added strength or stiffness from longitudinal jacket action. For thin carbon jackets wound directly onto the original column geometry, this gap can be very small, i.e., less than 25 mm (1 in), whereas in cases where concrete bolsters are added to convert column cross-sections to circular or oval shapes, gaps of 50 mm (2 in) or more can be required to prevent contact between the retrofit and the adjacent bent portions.

Since the principal deficiency in existing pre-1971 bridge columns is in the amount and detailing of the transverse reinforcement, the automated continuous carbon fiber wrap system addresses this deficiency by wrapping prepreg 12K carbon tows in the horizontal or 90° direction to the column axis, to provide the required transverse confinement, clamping and buckling restraints. Anchorage of the wound carbon tows is ensured by the continuity of the fiber wrap for the entire column jacket and lay-up thicknesses can be closely controlled and monitored with an automated winding system.

The key mechanical properties of the carbon jacket system, to provide confinement, clamping and buckling restraints, are the elastic jacket modulus E_j in the hoop direction, the ultimate unidirectional tensile strength f_{ju} and the ultimate unidirectional tension failure strain ε_{ju}. Since essentially linear elastic mechanical characteristics can be assumed for the unidirectional composite fiber wrap, two of the three characteristic properties are sufficient for the jacket design. The design guidelines outlined in the following can be applied to other composite fiber jacket systems with different material characteristics. However, appropriate reduction factors to the mechanical characteristics need to be defined for durability, non-uniformity in lay-up in case non-automated systems are used, non-continuous fibers or jacket joints in the hoop direction, and for systems where ambient curing rather than controlled curing environments are used.

2. SHEAR STRENGTH RETROFIT

2.1 Shear Mechanism

Many different models exist to describe the complex transfer of so called "shear forces" in a reinforced concrete member. A simple and rational model which seems to fit the experimental data best was put forward in [1] and assumes a combination of three different mechanisms to contribute to the nominal shear capacity V_n in the form:

$$V_n = V_c + V_s + V_p \tag{1}$$

where V_c = the concrete contribution provided primarily in the form of aggregate interlock, which decreases with increasing crack width and flexural ductility, V_s = the horizontal reinforcing steel contribution as part of an assumed truss mechanism, and V_p = the horizontal component from the applied axial load compression strut between the column ends.

Due to the aggregate interlock degradation with increasing crack width or flexural ductility, the V_c components needs to be tied to the column displacement ductility level μ_Δ in regions where inelastic flexural plastic hinging occurs. Thus, the concrete contribution to the shear resistance needs to be assessed both inside the plastic hinge region L_c (Figs. 5 and 6) as $V_c^{\,i}$ and outside the plastic hinge region over L_v (Figs. 5 and 6) as $V_c^{\,o}$. Thus

$$\text{a)}\quad \frac{V_c'}{N}=0.083k\sqrt{\frac{f_c'}{MPa}}\frac{A_e}{mm^2} \quad \text{or} \quad \frac{V_c'}{lbs}=k\sqrt{\frac{f_c'}{psi}}\frac{A_e}{in^2}$$

$$\text{b)}\quad \frac{V_c'}{N}=0.25\sqrt{\frac{f_c'}{MPa}}\frac{A_e}{mm^2} \quad \text{or} \quad \frac{V_c'}{lbs}=3\sqrt{\frac{f_c'}{psi}}\frac{A_e}{in^2} \tag{2}$$

where the effective concrete shear transfer area $A_e = 0.8\,A_g$ or 80 % of the gross column area, and k is a strength reduction factor based on the column displacement ductility μ_Δ in the form of

$$\begin{aligned} k &= 3 && \text{for } \mu_\Delta < 2 \\ k &= 5 - \mu_\Delta && \text{for } 2 \le \mu_\Delta < 4 \\ k &= 1.5 - \mu_\Delta / 8 && \text{for } 4 \le \mu_\Delta < 8 \\ k &= 0.5 && \text{for } \mu_\Delta \ge 8, \end{aligned} \tag{3}$$

a design relationship put forward in [2] for unidirectional ductility, which can also be graphically expressed as shown in Fig. 7. Note that Eq. (3) is for shear design and is thus slightly more conservative than concrete shear reductions proposed for assessment of expected capacities in existing columns (Fig. 7).

The horizontal reinforcing steel contribution V_s can be determined as

$$\begin{aligned} &\text{a)}\quad V_s = \frac{\pi}{2}\frac{A_h f_{hy} D'}{s}\cot\theta \quad \text{(circular)} \\ &\text{b)}\quad V_s = \frac{n A_h f_{hy} D'}{s}\cot\theta \quad \text{(rectangular)} \end{aligned} \tag{4}$$

where A_h = the area of one leg of the horizontal reinforcement, n = the number of legs of horizontal column ties in the loading direction, f_{hy} = the yield strength of the horizontal reinforcement, s = the spacing of the horizontal reinforcement or the spiral pitch, θ = the angle of the principal compression strut to the column axis or the shear crack inclination, and D' = the core column dimension in the loading direction from center to center of the peripheral horizontal reinforcement (Fig. 8). Conservatively, $\theta = 45°$ or $\cot\theta = 1$ can be assumed for design, or more accurately, for assessment, principal compression strut inclinations of 30° can be assumed for bridge columns and 45° for pier walls.

The axial load shear contribution is simply defined as the horizontal component of the inclined compression strut:

$$V_p = P\tan\alpha \tag{5}$$

where P represents the axial load at the column top and α the compression strut inclination or angle with the vertical column axis. This tan α can be defined as

$$\text{a)}\quad \frac{D-c}{2L} \qquad \text{(for single bending)}$$

$$\text{b)}\quad \frac{D-c}{L} \qquad \text{(for double bending)}$$

where c represents the distance between the neutral axis and the extreme compression fiber, D the column dimension in the loading direction, and L the clear column height, as depicted in Fig. 9.

2.2 Carbon Jacket Shear Retrofit

Carbon jackets of thickness t_j contribute an additional or fourth term to the shear resistance mechanism outlined in Eq. (1) in the form

$$\begin{aligned} &\text{a)}\quad V_j = \frac{\pi}{2} f_{jd} t_j D \cot\theta \quad \text{(circular)} \\ &\text{b)}\quad V_j = 2 f_{jd} t_j D \cot\theta \quad \text{(rectangular)} \end{aligned} \qquad (6)$$

where t_j = the carbon jacket thickness, f_{jd} = the design stress level in the jacket, and D = the column dimension in the loading direction. Again, conservatively, a 45° force transfer mechanism, or cot θ = 1 can be assumed for the jacket design.

While Eq. (6) clearly indicates that the jacket contribution depends on the jacket stress, a stress level less than the ultimate capacity f_{ju} is assumed to limit the horizontal column dilation. Tests at UCSD [1,5] have shown that, when the column dilation exceeds 0.4 % to 0.5 % in the loading direction, the concrete contribution to the shear capacity degrades rapidly, thus a strain limit rather than a strength limit needs to be employed for the jacket design. A strain limit of ε_{jd} = 0.4 % is a conservative design value, which is well below the ultimate strain limit of $\approx$ 1 % for the carbon jacket but higher than the yield strain of the horizontal column reinforcement which will ensure that the column reinforcement shear contribution in Eq. (4) will be fully activated. Thus, in Eq. (6)

$$f_{jd} = 0.004\, E_j \qquad (7)$$

should be used for the composite jacket shear design.

2.3 Shear Retrofit Design

The carbon jacket shear retrofit design can be summarized as follows. The shear design demand originates, based on capacity design principles, from the plastic column shear or the shear at full overstrength V_0. With a shear strength reduction factor ϕ = 0.85 the column shear design requires that

$$V_n = V_c + V_s + V_p + V_j \geq \frac{V_o}{\phi} \qquad (8)$$

Unless more reliable actual plastic shear information is available, V_o can be conservatively estimated as 1.5 V_{yi} or 1.5 times the ideal shear capacity of the column at ductility $\mu_\Delta = 1$, or

$$V_j \geq \frac{V_o}{\phi} - (V_c + V_s + V_p) \tag{9}$$

For circular columns the jacket thickness t_j can be determined as

$$t_j = \frac{\frac{V_o}{\phi} - (V_c + V_s + V_p)}{\frac{\pi}{2} 0.004 E_j D} = \frac{159}{E_j D}\left[\frac{V_o}{\phi} - (V_c + V_s + V_p)\right] \tag{10}$$

and for rectangular columns

$$t_j = \frac{\frac{V_o}{\phi} - (V_c + V_s + V_p)}{2 \times 0.004 E_j D} = \frac{125}{E_j D}\left[\frac{V_o}{\phi} - (V_c + V_s + V_p)\right] \tag{11}$$

Since the concrete shear contribution V is different inside the plastic hinge confinement region (L_c) and outside (L_v), two jacket thicknesses for shear have to be derived and provided over the regions L_v^i and L_v^o in Figs. 5 and 6, respectively. To avoid shear problems within and in direct vicinity to the flexural plastic hinge, the shear retrofit length L_v^i should be extended to $L_v^i = 1.5$ D or one and a half times the column dimension in the loading direction measured from the point of maximum moment.

3. FLEXURAL PLASTIC HINGE CONFINEMENT

3.1 Flexural Plastic Hinge Mechanism for Circular Columns

Confinement of flexural plastic hinge regions in columns is required to enhance the ultimate compression strain of the concrete and with it, the inelastic rotation capacity of the hinge, as well as to support the longitudinal reinforcement against lateral buckling.

To confine a flexural plastic hinge region in a circular column for standard design ductilities a volumetric reinforcement ratio of

$$\rho_s = \frac{k_s f'_c}{f_{yh}}\left[0.5 + 1.25\frac{P}{f'_c A_g}\right] + 0.13(\rho_\lambda - 0.01) \tag{12}$$

is required based on [2, 3] in the plastic hinge zone. Equation (12) depends on ρ_ℓ = the longitudinal column reinforcement ratio A/A_g and on a factor k_s which is calibrated with experimental results based on an energy balance approach which compares the vertical strain energy stored in the confined concrete at crushing with the strain energy stored provided by the horizontal hoop reinforcement up to bar rupture. For mild steel reinforcement hoops, $k_s = 0.16$ is applied [2, 3].

For carbon fiber jackets Eq. (12) can be interpreted as

$$\rho_j = \frac{4t_j}{D} = \frac{k_j f_c'}{f_{ju}}\left[0.5 + 1.25\frac{P}{f_c' A_g}\right] + 0.13(\rho_\lambda - 0.01) \tag{13}$$

Based on energy considerations as outlined above, the characteristic hoop reinforcement strain energy for elastoplastic stress-strain characteristics of mild steel in the form of $[f_{hy}\ \varepsilon_{hu}]$ can be expressed for carbon jackets with essentially linear elastic stress-strain characteristic in the form of $[\frac{1}{2} f_{ju}\ \varepsilon_{hu}]$, which, for typical mild steel ($f_y = 455$ MPa or 66 ksi, $\varepsilon_{su} = 15$ %) and unidirectional tows ($f_{ju} = 1\ 380$ MPa or 200 ksi, $\varepsilon_{su} = 1$ %) would result in an efficiency reduction to approximately 10 % for the carbon jacket due to the low strain limits. However, tests on carbon fiber jacketed columns at UCSD [7 to 12] have shown that significantly higher compression strains (by a factor of 3 to 4) can be achieved in the confined concrete than predicted by the energy balance approach, which can be attributed to the reduced concrete dilation due to the lower ultimate strain limits in the carbon jacket. Thus, for carbon jacket retrofit designs, the confinement efficiency can conservatively be increased by at least a factor of two, resulting in an equivalent confinement factor of

$$k_j = \frac{k_s}{2 \times 0.1} = 5k_s = 0.8 \tag{14}$$

or a carbon jacket thickness

$$t_j = \frac{D}{5}\frac{f_c'}{f_{ju}}\left[0.5 + 1.25\frac{P}{f_c' A_g}\right] + 0.13(\rho_\lambda - 0.01) \tag{15}$$

The ultimate compression strain in the confined concrete can be expressed based on [4] as

$$\varepsilon_{cu} = 0.004 + \frac{2 \times 1.4 \rho_j f_{ju} \varepsilon_{ju}}{f_{cc}'} \tag{16}$$

where f_{cc}' = the compression strength of the confined concrete conservatively taken as 1.5 f_c' and the factor 2 again represents the conservative estimate of increased compression strains based on the observed experimental data from carbon fiber jacket confined plastic hinges.

With this ultimate concrete strain and a depth c_u for the flexural compression zone calculated as part of normal flexural strength calculations or from a moment curvature analysis, the resulting ultimate curvature

$$\phi_u = \frac{\varepsilon_{cu}}{c_u} \tag{17}$$

can be determined. Together with the ϕ_y from an equivalent bilinear moment-curvature approximation (obtained from the moment-curvature section analysis), the curvature ductility

$$\mu_\phi = \frac{\phi_u}{\phi_y} \tag{18}$$

can be determined, which in turn can be expressed in the form of a member ductility factor

$$\mu_\Delta = 1 + 3(\mu_\phi - 1)\frac{L_p}{L}\left(1 - 0.5\frac{L_p}{L}\right) \tag{19}$$

where L_p = the equivalent plastic hinge length defined as

$$L_p = 0.8L + 0.022\frac{f_{sy}}{MPa}d_b \quad \text{or} \quad L_p = 0.8L + 0.15\frac{f_{sy}}{ksi}d_b \tag{20}$$

where f_{sy} = the yield strength and d_b = the bar diameter of the longitudinal column reinforcement. The length L is again defined in Figs. 5 and 6. Note that Eq. (20) is the same as the one used to assess unretrofitted columns since 90°carbon fiber wraps do not contribute to longitudinal column capacities or provide restrictions to the plastic hinge development.

Alternatively, for a given ε_{cu} which can be directly derived based on design ductility requirements from back calculation of Eqs. (19) to (16), the required jacket thickness can be expressed as

$$t_j = \frac{\rho_j D}{4} = 0.09\frac{D(\varepsilon_{cu} - 0.004)f'_{cc}}{f_{ju}\varepsilon_{ju}} \tag{21}$$

which generally results in a more economical jacket thickness than required by the standard confinement ratio Eqs. (12, 15).

To prevent column bar buckling in the plastic hinge region [2, 3], a volumetric transverse reinforcement ratio of

$$\rho_s = \frac{0.45 n f_s^2}{E_{ds} E_t} \tag{22}$$

is required, where

$$E_{ds} = \frac{4 E_s E_i}{\left(\sqrt{E_s} + \sqrt{E_i}\right)^2} \tag{23}$$

and E_t = the modulus of elasticity of the transverse reinforcement, n = the number of longitudinal reinforcing bars, f_s = steel stress at a strain of 4 % in the longitudinal reinforcement or 510 MPa (74 ksi) for grade 60 steel, E_s = the secant modulus from f_s to f_u, and E_i = the initial elastic modulus of the longitudinal reinforcement. For longitudinal grade 60 steel, E_{ds} can thus be determined as 4 530 MPa (657 ksi), resulting in

$$\rho_s = \frac{25.86\,n}{\frac{E_t}{MPa}} \quad \text{or} \quad \rho_s = \frac{3.75\,n}{\frac{E_t}{ksi}} \tag{24}$$

For a carbon fiber jacket Eq. (24) can be expressed as

$$\rho_s = \frac{4t_j}{D} = \frac{25.86\,n}{\frac{E_j}{MPa}} \quad \text{or} \quad \rho_s = \frac{4t_j}{D} = \frac{3.75\,n}{\frac{E_j}{ksi}} \tag{25}$$

The anti-buckling requirement of Eq. 25 only needs to be checked for slender columns where L^i, the distance between maximum moment location and point of inflection (Figs. 5 and 6) is greater than 4D, i.e., M / (VD) > 4.

All of the above considerations apply to circular columns.

3.2 Rectangular Columns

In cases where oval jackets can be employed on oblong or rectangular columns, an equivalent column diameter D_e

$$D_e = R_1 + R_3 \tag{26}$$

can be employed with jacket radii defined as

$$R_1 = \frac{b^2}{a}\ , \ R_3 = \frac{a^2}{b} \tag{27}$$

with a and b the oval jacket principal dimensions, as shown in Fig. 10.

For rectangular column side dimensions A and B (Fig. 10), the oval jacket dimensions a and b which minimize the total length of principal axes for an elliptical jacket can be found as

$$\begin{aligned} a &= k\ b \\ b &= \sqrt{\left(\frac{A}{2k}\right)^2 + \left(\frac{B}{2}\right)^2} \\ k &= \left(\frac{A}{B}\right)^{\frac{2}{3}} \end{aligned} \tag{28}$$

The effectiveness of confining rectangular columns with rectangular jackets decreases significantly since only corner forces are generated during the dilation of the column flexural hinge. Tests on rectangular columns at UCSD [10] retrofitted with rectangular carbon jackets indicated a jacket efficiency of only 50 % of that provided by a circular jacket or an oval jacket with the above defined equivalent radius. However, only column side aspect ratios of 1.5 were tested. Thus, for columns with side aspect ratios of 1.5 or less, a jacket thickness of twice the one calculated for an equivalent circular jacket should be assigned, whereas for columns with aspect ratios > 1.5 extrapolation of the test results to date is not recommended and oval or circular jackets should be designed.

3.3 Design of Flexural Confinement Retrofits

For confinement of flexural plastic hinge regions where the ultimate jacket stress controls the design, a long-term durability strength reduction factor of 0.9 should be employed for the carbon jacket design. For other composite materials appropriate strength reduction factors based on their expected durability characteristics should be assigned.

a) Circular Columns:

For circular columns with column diameter D, longitudinal reinforcement ratio ρ_ℓ, expected concrete strength f_c', axial load P, gross section area A_g, and ultimate jacket modulus f_{ju}, the carbon jacket thickness t_j can be determined as

$$t_j = \frac{D}{4.5}\frac{f_c'}{f_{ju}}\left[0.5 + 1.25\frac{P}{f_c' A_g}\right] + 0.13(\rho_\lambda - 0.01) \tag{29}$$

The resulting member ductility should be checked based on Eqs. (16) to (19).

Alternatively, for a given member ductility μ_Δ and required ultimate concrete strain ε_{cu}

$$t_j = 0.09\frac{D(\varepsilon_{cu} - 0.004)f_{cc}'}{0.9 f_{ju}\varepsilon_{ju}} = 0.1\frac{D(\varepsilon_{cu} - 0.004)f_c'}{f_{ju}\varepsilon_{ju}} \tag{30}$$

can be provided.

To prevent column bar buckling for columns with shear span L/D = M/(VD) > 4, a minimum jacket thickness of

$$t_j = \frac{6.9\,n\,D}{\dfrac{E_j}{\text{MPa}}} \quad \text{or} \quad t_j = \frac{n\,D}{\dfrac{E_j}{\text{ksi}}} \tag{31}$$

should be provided.

b) Rectangular Columns

For side aspect ratios ≤1.5, rectangular columns can be retrofitted for flexural confinement with rectangular jackets under the following design considerations:

(1) the corners need to be rounded to a radius of ≥50 mm (2 in) (25 mm or 1 in was used in the laboratory tests)

(2) the jacket thickness t_j should be twice that determined from a column with equivalent circular diameter D_e, where D_e is determined from Eqs. (26) to (28).

In all other cases where the side aspect ratio > 1.5, oval or circular carbon jackets should be designed by adding oval or circular concrete segments to the bridge column sides prior to wrapping and curing.

c) Extent of Flexural Hinge Confinement Retrofit

The jacket thickness t_j must be extended beyond the expected plastic hinge region. For bridge columns with typical axial load ratios $P/(f_c' A_g) \leq 0.3$, the confinement length L_{c1} should be greater than L/8 and greater than 0.5 D (Figs. 5 and 6) measured from the location of maximum moment. In addition, a reduced jacket thickness of 0.5 t_j should be extended for a distance L_{c2} defined by the same criteria as L_{c1} but starting at L_{c1}.

Furthermore, where jackets and/or concrete bolsters add significantly to the column dimension in the loading direction, a gap g between the retrofit measure and the adjacent bridge bent member (cap or footing) needs to be provided to avoid any strength and stiffness increase from the retrofit. For most bridge columns and retrofits, a gap of 50 mm (2 in) is sufficient to meet this objective. Other gap widths can be explicitly calculated based on the maximum expected hinge rotation and column bar buckling considerations.

4. CLAMPING OF LAP SPLICES

4.1 Lap Splice Failure Mechanism

A simplified failure model developed by Priestley [5, 6] assumes that lap splice debonding occurs in the form of failure planes in the cover concrete and along the longitudinal column bar perimeter as outlined in Fig. 11. The postulated failure model assumes the pull-out of concrete prisms. To restrain this concrete prism pull-out, clamping forces across the debonding interface and the concept of shear friction with a friction coefficient of μ = 1.4 for naturally occurring concrete cracks can be assumed.

Based on the circular jacket confinement model in Fig. 12, the jacket tensile force T_j is developed by the jacket stress f_j acting over the jacket thickness t_j as

$$T_j = t_j f_j \tag{32}$$

Equilibrium of forces with an internal lateral or dilation pressure f_ℓ, can be obtained by

$$2 t_j f_j = f_\lambda D \tag{33}$$

and the required jacket thickness can be defined as

$$t_j = \frac{D f_\lambda}{2 f_j} \tag{34}$$

as a function of the lateral clamping pressure f_ℓ required to keep the lap splice reinforcement from debonding.

The debonding criteria can be obtained from the model in Fig. 11 as

$$f_\lambda = \frac{A_s f_{sy}}{\left(\frac{p}{2n} + 2(d_b + cc)\right) L_s} \tag{35}$$

where A_s = the area of one longitudinal reinforcing bar, f_{sy} = the yield strength of the longitudinal reinforcement, p = the inside crack perimeter along the longitudinal column reinforcement, n = the number of bars, d_b = the bar diameter, cc = the concrete cover to the longitudinal column reinforcement, and L_s = the lap splice length. Equation (35) assumes a 40 % overstrength of the column reinforcement past the yield stress level f_{sy} or that 1.4 f_{sy} needs to be developed in the lap splice.

Strain measurements [1] of the hoop strains in the carbon fiber jacket over the lap splice region showed that slip of the lap splice or lap splice debonding started at hoop strain levels between 0.001 and 0.002 . Clearly at strain levels above 0.002 debonding was in progress as indicated by gradual loss of lateral load carrying capacity. Setting ε_j = 0.001 as the design limit state to prevent lap splice debonding, the jacket stress f_{jd} in Eq. (34) has to be limited to

$$f_{jd} = E_j \varepsilon_j = 0.001 E_j \tag{36}$$

In columns with low transverse reinforcement ratios, the contribution to the lateral clamping force by the horizontal reinforcement is typically ignored. This applies particularly to columns with non-circular ties since only the inner bars or tied bars would benefit from the clamping force. In cases where circular hoops or spirals are present their contribution to the lateral clamping force can be evaluated at the same dilation strain limit of ε_d = 0.001 as

$$f_h = \frac{0.002 A_h E_h}{D s} \tag{37}$$

where A_h = the area of the hoop or spiral reinforcement, E_h = the horizontal reinforcement modulus, D should be the spiral or hoop inside diameter but can be closely approximated by the column diameter, and s = the hoop or spiral spacing, unless volumetric reinforcement ratios $\rho_{vh} > 0.5$ % are provided.

4.2 Lap Splice Clamping Design

Based on the above mechanisms and the jacket strain limit of Eq. (36), the jacket thickness can be obtained from Eq. (34), or can be found from

$$t_j = 500 \frac{D(f_\lambda - f_h)}{E_j} \tag{38}$$

Equation (38) applies to circular columns.

Since the lateral confinement pressure f_ℓ to prevent lap splice debonding can be quite high (up to and greater than 2.0 MPa [300 psi]) the convex jacket curvature is needed to provide this clamping force. Thus, no rectangular column wraps are recommended at this stage to prevent lap splice debonding. However, if controlled debonding is permissible, rectangular jackets can prevent the cover concrete from spalling and preserve the vertical or gravity load carrying capacity of the column. Again, a design rule similar to the plastic hinge confinement in terms of twice the jacket thickness for circular columns is recommended with the same limitations on column side aspect ratios.

For all other column geometries and cases where lap splice debonding is to be prevented, a circular or oval jacket with appropriate concrete bolsters needs to be provided. The jacket design follows Eq. (37) with an equivalent diameter D as defined in Eq. (26) and with the contribution from horizontal stirrups ignored, i.e., $f_h = 0$ in Eq. (37).

The lap splice retrofit should extend over the lap length L_s as indicated in Figs. *5* and 6.

5. SUMMARY AND CONCLUSIONS

Design guidelines for continuously wound carbon jackets for bridge column retrofit were developed based on rational design models and existing and proven design and retrofit principles involving steel jackets. Separate design criteria for (1) shear strengthening, (2) flexural plastic hinge confinement, and (3) lap splice clamping were developed.

For shear retrofit, separate design criteria for circular and rectangular column jackets were derived. The same approach can also be applied for pier walls. In the cases of flexural plastic hinging and lap splice clamping, the jacket design criteria were developed for circular columns and recommendations are provided for column side aspect ratios for which rectangular carbon jackets can also be employed. The experimentally verified range of column side aspect ratios is $D/B \leq 1.5$. For these aspect ratios, rectangular carbon jacket retrofits with twice the jacket thickness developed for a circular column with an equivalent column diameter should be employed, since only a 50 % effectiveness of the rectangular jacket confinement was observed. Lap splice debonding cannot effectively be prevented with rectangular column jackets and the columns need to be converted to oval or circular cross-sections prior to retrofit application.

6. REFERENCES

[1] Priestley, M.J.N, Seible, F. Verma, R. and Xiao, Y., "Seismic Shear Strength of Reinforced Concrete Columns," Structural Systems Research Project, Report No. SSRP-93/06, University of California, San Diego, La Jolla, CA, July 1993, 120 pp.

[2] Priestley, M.J.N, Seible, F. and Calvi, M., "Seismic Design and Retrofit of Bridges," Wiley & Sons, New York, 1995.

[3] ATC-32, Revised BDS, May 1995

[4] Mander, J.B., Priestley, M.J.N. and Park, R., "Theoretical Stress-Strain Model of Confined Concrete," ASCE Journal of Structural Engineering, Vol. 114, No. 8, August 1988, pp. 1827-1849.

[5] Priestley, M.J.N, and Seible, F. (editors), "Seismic Assessment and Retrofit of Bridges," Structural Systems Research Project, Report No. SSRP-93/06, University of California, San Diego, La Jolla, CA, July 1991, 426 pp.

[6] Priestley, M.J.N, Seible F. and Chai, Y.H., "Design Guidelines for Assessment Retrofit and Repair of Bridges for Seismic Performance," Structural Systems Research Project, Report No. SSRP-92/Ol. University of California, San Diego, La Jolla, CA, August 1992, 266 pp.

[7] Seible, F., Hegemier, GA., Priestley, M.J.N., Innamorato, D., Weeks, J., and Policelli, F., "Carbon Fiber Jacket Retrofit Test of Circular Shear Bridge Column, CRC-2," Advanced Composites Technology Transfer Consortium Report No. ACTT-94/02, UCSD, September 1994, 49 pp.

[8] Seible, F., Hegemier, GA., Priestley, M.J.N., and Innamorato, D., "Seismic Retrofitting of Squat Circular Bridge Piers with Carbon Fiber Jackets," Advanced Composites Technology Transfer Consortium Report No. ACTT-94104, UCSD. November 1994, 55 pp.

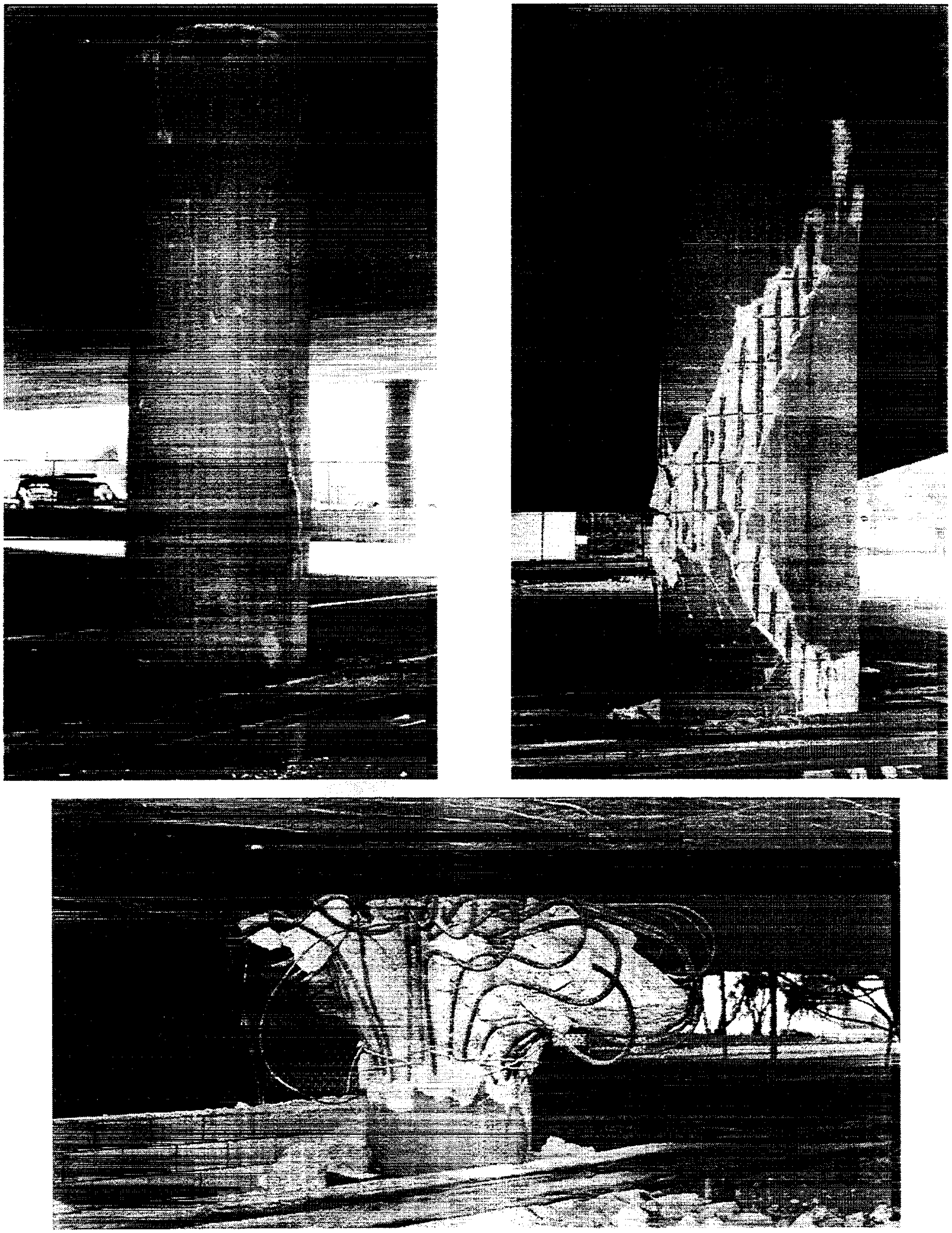
[9] Seible, F., Hegemier, G.A., Priestley, M.J.N., Innamorato, D., and Ho, F., "Carbon Fiber Jacket Retrofit Test of Rectangular Flexural Column with Lap Spliced Reinforcement" Advanced Composites Technology Transfer Consortium Report No. ACTT-95/02, UCSD, March 1995, 53 pp.

[10] Seible, F., Hegemier, G.A., Priestley, M.J.N., Ho, F., and Innamoraro, D., "Rectangular Carbon Jacket Retrofit of Flexural Column with 5 % Continuous Reinforcement," Advanced Composites Technology Transfer Consortium Report No. ACTT-95/03, UCSD, April 1995, 52 pp.

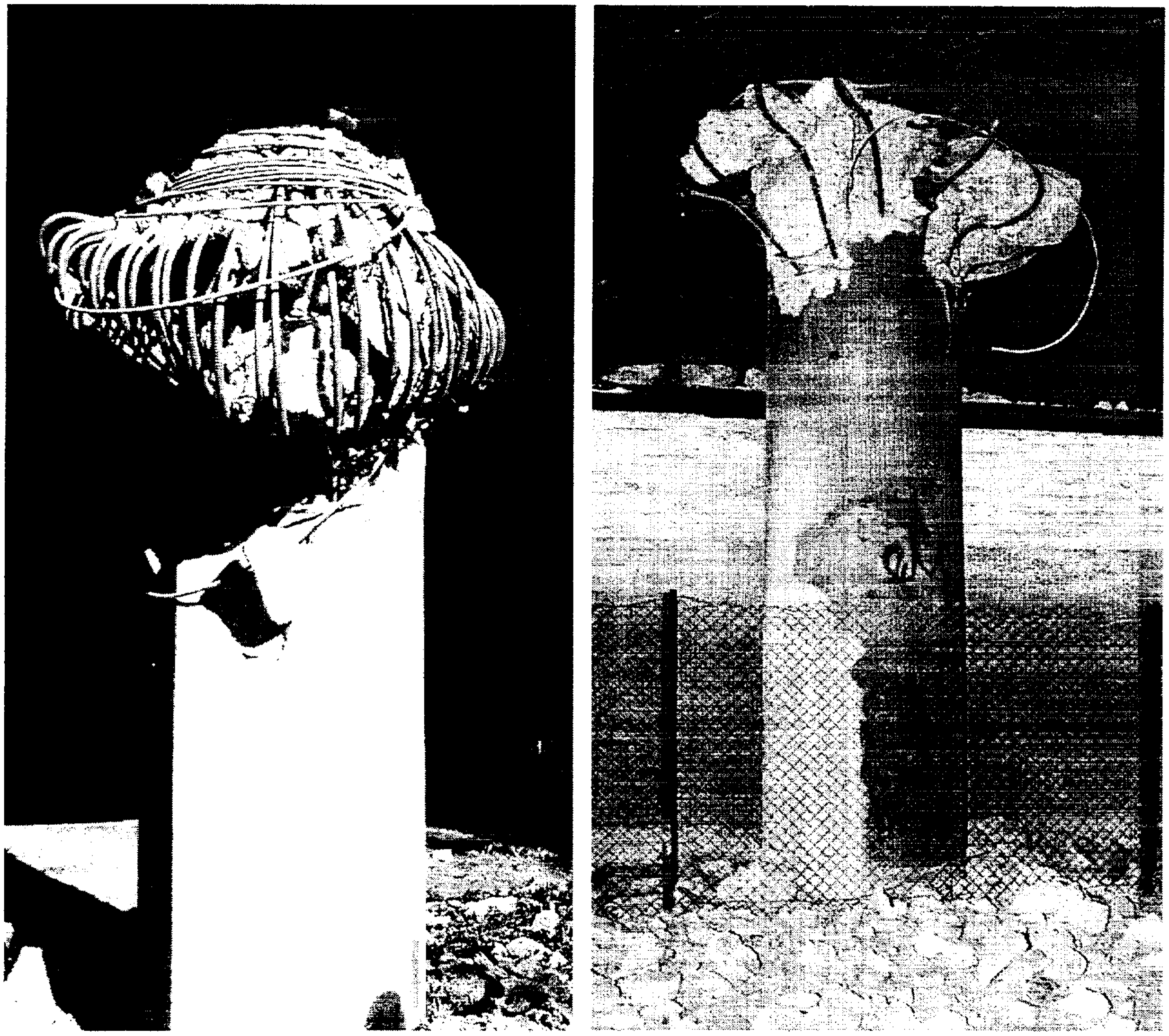
[11] Seible, F., Hegemier, G.A., Priestley, M.J.N., Innamorato, D., and Ho, F., "Carbon Fiber Jacket Retrofit Test Of Circular Flexural Columns With Lap Spliced Reinforcement," Advanced Composites Technology Transfer Consortium Report No. ACTT-95/04, UCSD, June 1995, 78 pp.

[12] Seible, F., Hegemier, G.A., Priestley, M.J.N., and Innamorato, D., "Rectangular Carbon Fiber Jacket Retrofit Test Of A Shear Column With 2.5 % Reinforcement," Advanced Composites Technology Transfer Consortium Report No. ACTT-95/05, UCSD, July 1995, 50 pp.

[13] Policelli, F., "Carbon Fiber Jacket Wrapping of Five Columns on the Santa Monica Viaduct, Interstate 10, Los Angeles," Advanced Composites Technology Transfer/Bridge Infrastructure Renewal Consortium. Report No. ACTT/BIR-95/14, August 1995, 110 pp.



FGI. 1. Progressive Shear Failure, I-10 Santa Monica Freeway, Northridge Earthquake 1994



a) SR-118 Bull Creek Channel Bridge

b) I-10 La Cienega and Venice

FIG. 2. Flexural Plastic Hinge Failures, Northridge Earthquake 1994

a) As-Built

b) Steel Jacket Retrofit ($\mu_\Delta = 7.7$)

c) Carbon Fiber Retrofit ($\mu_\Delta = 8$)

FIG. 3. Flexural Hinge Failure and Retrofits of 5 % Reinforced Rectangular Column

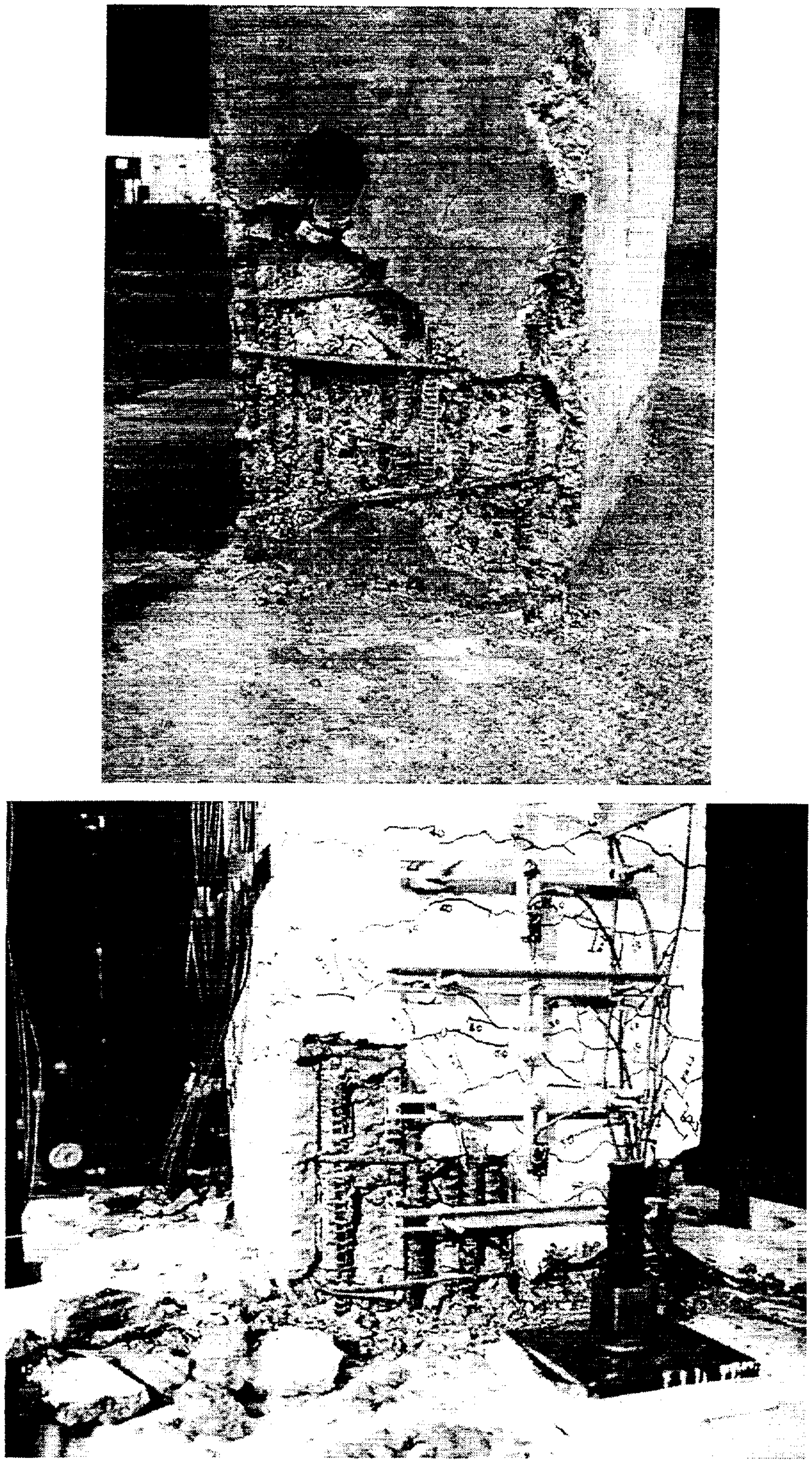


FIG. 4. Lap Splice Bond Failure During Loma Prieta 1989 and in the 40 % Scale Laboratory Tests

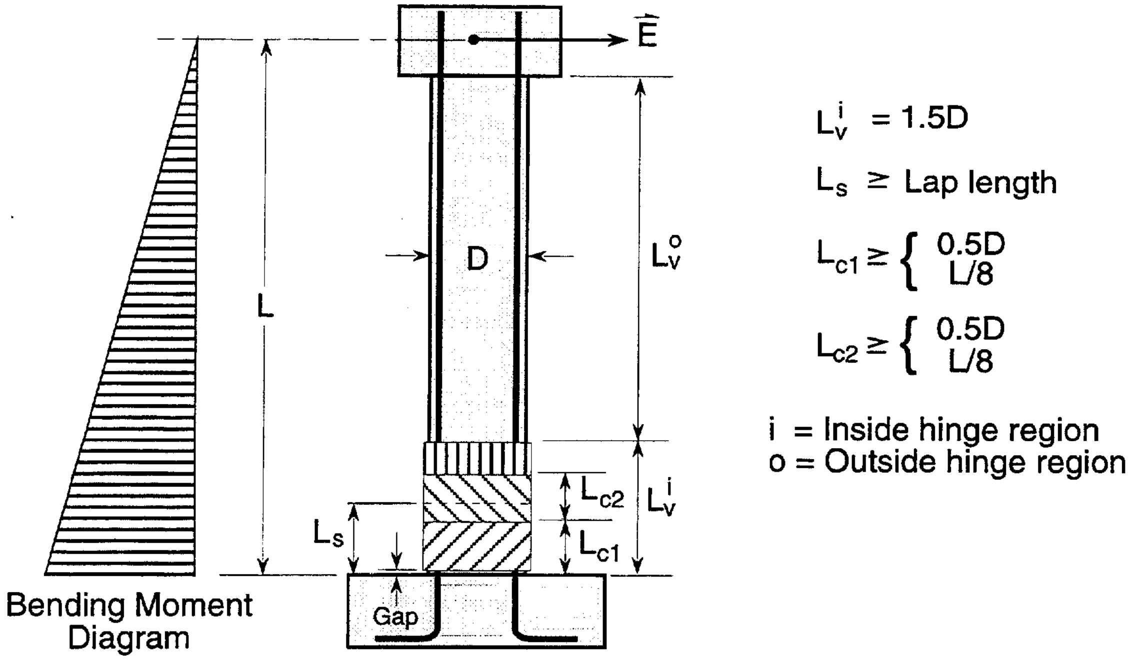


FIG. 5. Carbon Jacket Regions for Bridge Column Retrofit, Single Bending

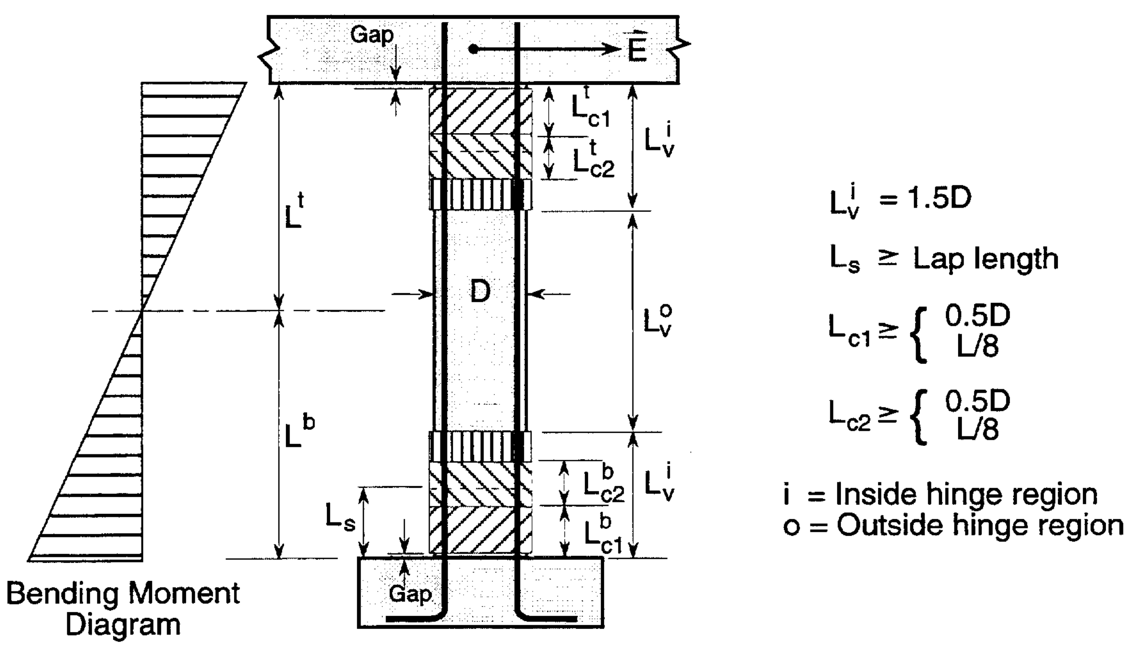


FIG. 6. Carbon Jacket Regions for Bridge Columns Retrofit, Double Bending

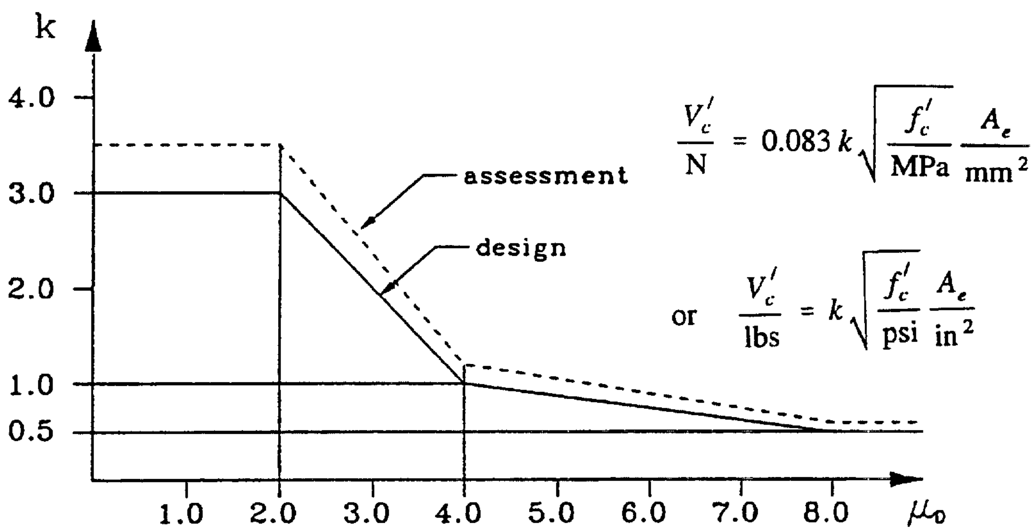
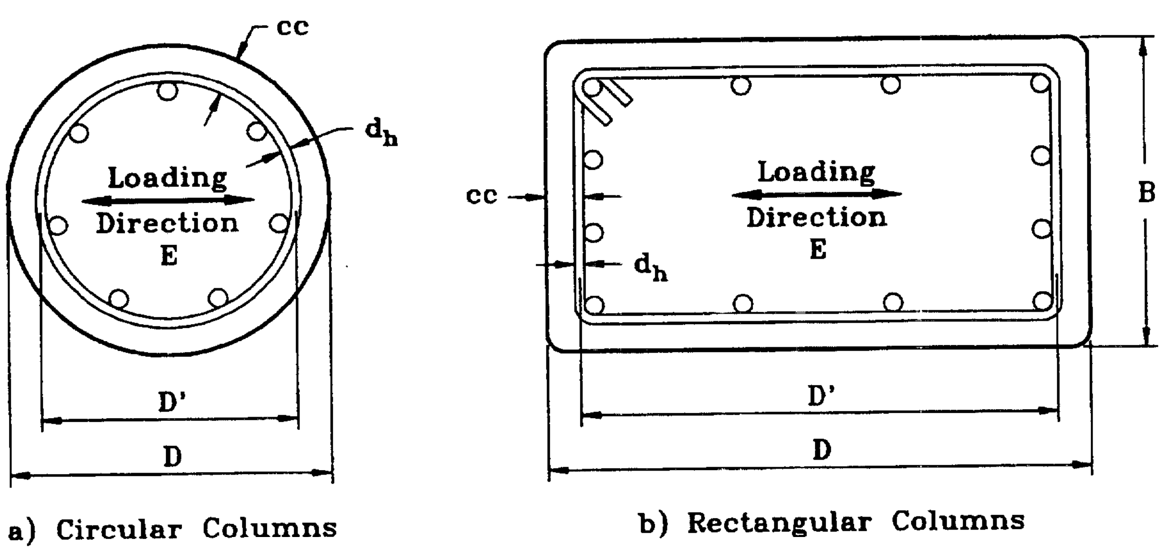


FIG. 7. Relationship Between Unidirectional Ductility and Design Concrete Shear Contribution



$$D' = D - 2cc + d_h$$

FIG. 8. Definition of Column Core Dimension D'

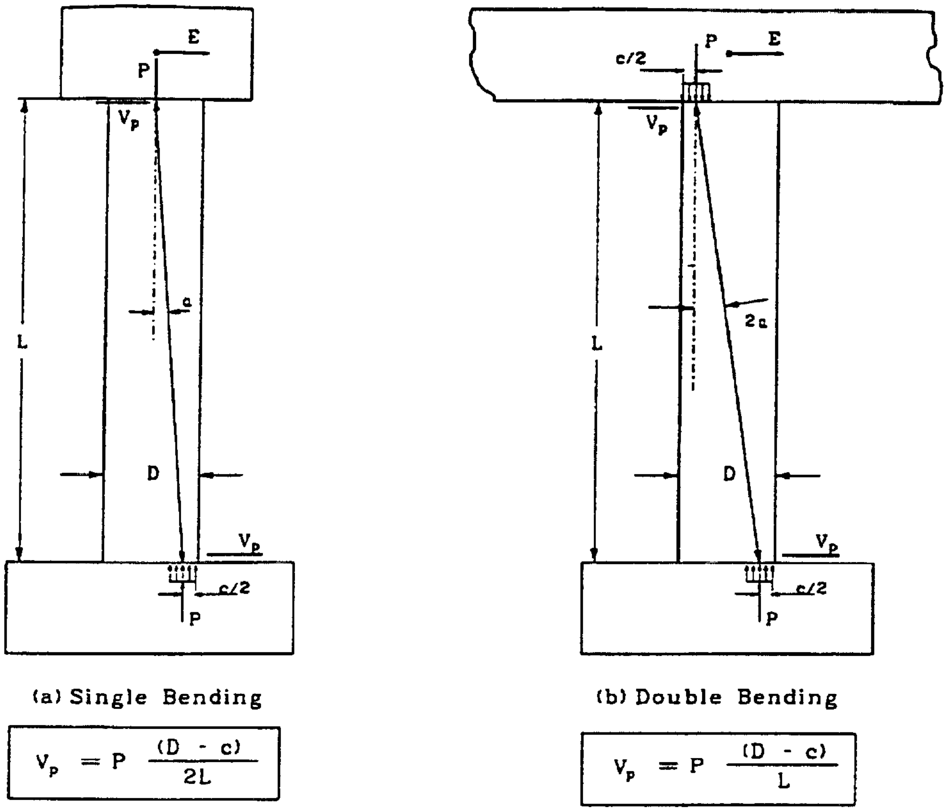


FIG. 9. Axial Force Shear Contribution

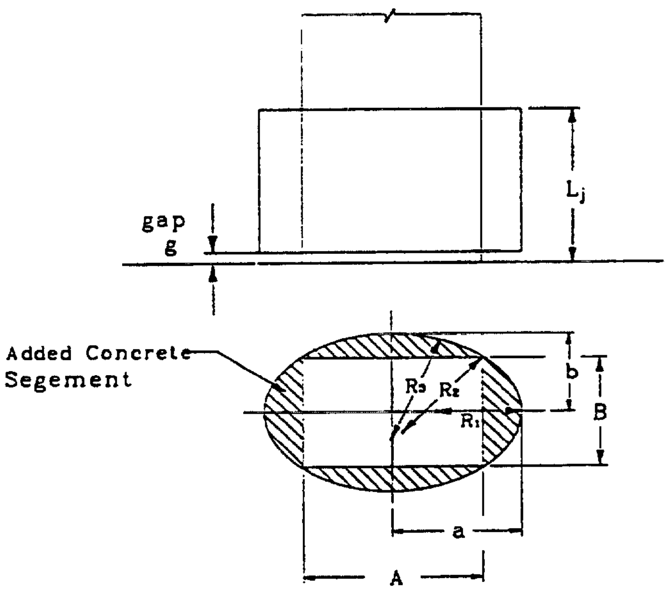


FIG. 10. Confinement of Rectangular Column Base with Oval Carbon Jacket and Added Concrete Segments

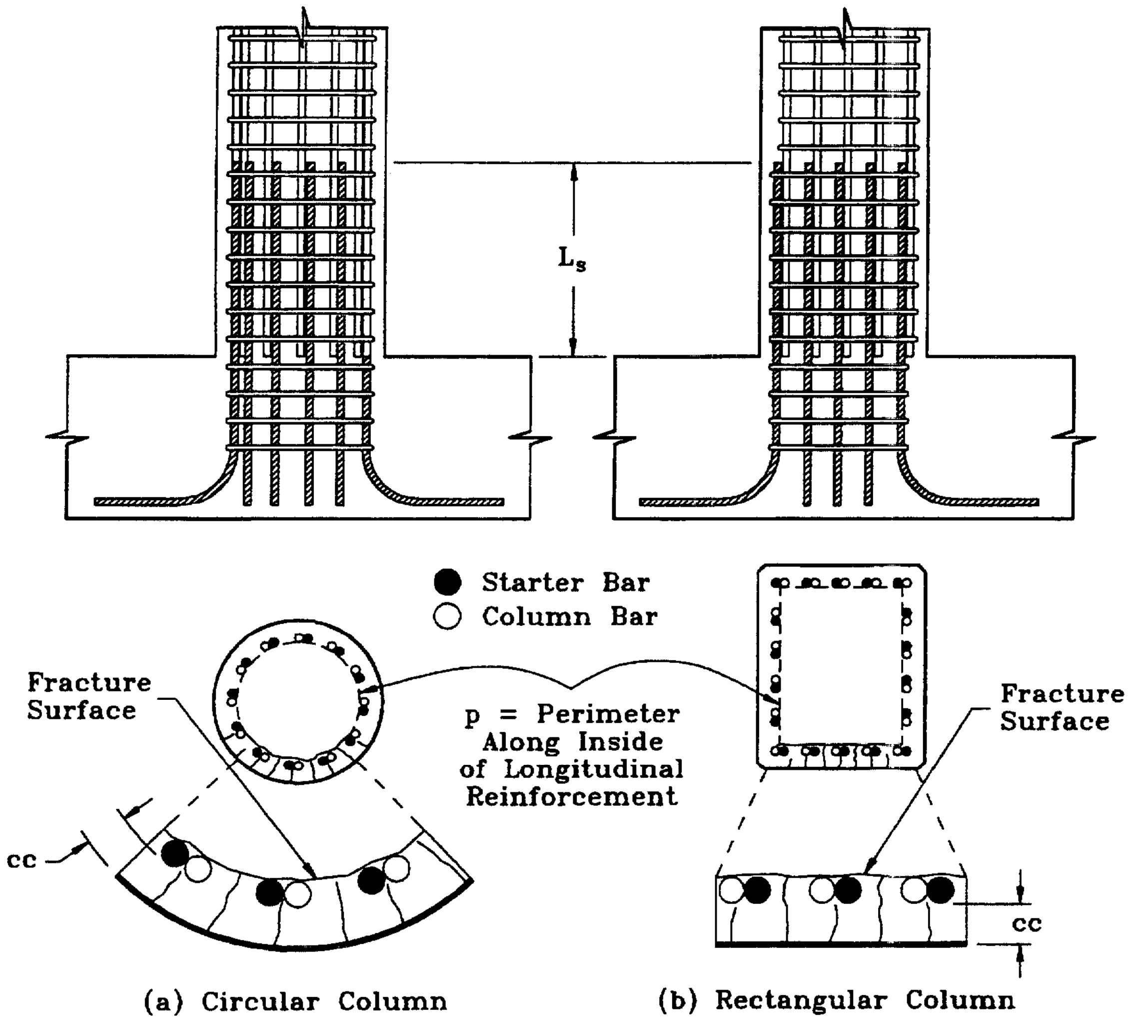


FIG. 11. Lap Splice Failure Model

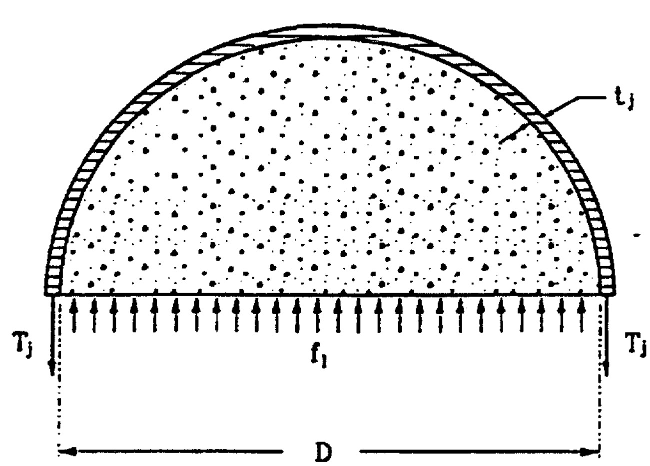


FIG. 12 Confinement of Circular Column by Passive Jacket

APPENDIX 3.1B

CALTRANS SPECIFICATIONS

In conformance with NIST policy, SI units are used as primary units in this document. U.S. customary units, used exclusively in the original specifications, are included in parentheses.